IN-FIELD SIMULATED SEISMIC TESTING OF AS-BUILT AND RETROFITTED UNREINFORCED MASONRY PARTITION WALLS OF THE WILLIAM WEIR HOUSE IN WELLINGTON

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ABSTRACT

Unreinforced masonry (URM) partition walls of William Weir House in Wellington were subjected to out-of-plane forces to investigate the as-built wall characteristic behaviour. The lateral load resisting system of the 1932 reinforced concrete building was scheduled to undergo seismic strengthening, and due to the absence of reliable wall out-of-plane assessment data, consulting engineers adopted an experimental proof-testing approach. A team of student researchers from the University of Auckland tested four URM partition walls by subjecting the walls to out-of-plane uniform pressure applied by means of a system of airbags. The testing included two mid-storey and two top-storey URM partitions, which had developed prior minor structural cracks. The full-scale in-situ testing confirmed that the pre-cracked partitions had sufficient strength to resist the current New Zealand seismic demand, and the experimental programme resulted in substantial financial benefits for the client as none of the walls were identified as demolition or strengthening candidates. In addition to the as-builtout-of-plane tests, two tests were conducted on partition walls retrofitted using near-surface-mounted (NSM) fiber-reinforced polymer (FRP) strips. The results of the as-built and the retrofitted wall testing are reported, the wall behaviour is evaluated against the current seismic demand and the assessment results are compared with the New Zealand Society for Earthquake Engineering (NZSEE) recommendations.

1.0 INTRODUCTION

Unreinforced masonry (URM) buildings are a class of structures that are built using masonry units with either lime or cement mortar, without the provision of supplementary reinforcement. Many URM buildings or building components in New Zealand are potentially “earthquake prone” according to the provisions of the Building Act 2004 (DBH 2004), which defines such structures as having a seismic capacity level that is less than one third of that required for a new building constructed at the same site. The New Zealand legislation requires earthquake prone buildings to be seismically upgraded or removed. This requirement is often accompanied by the heritage importance of the relatively old buildings (Goodwin 2009) and has prompted seismic evaluation and retrofit projects being undertaken throughout New Zealand.

Reports of past earthquakes have revealed that a high level of seismic risk is associated with existing URM walls when subjected to out-of-plane excitations. Wall out-of-

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plane collapse has been one of the most common forms of building damage observed during past earthquakes, and such damage jeopardizes the gravity-load carrying capacity of the building (Bruneau 1994). Blaikie and Spurr (1992) reported the impact of seven earthquakes on buildings and concluded that out-of-plane failure of URM building components was the most prominent form of building damage in the majority of events.

Derakhshan et al. (2008, 2009a, 2010a) reported the results of out-of-plane airbag tests on laboratory-constructed URM walls, and Derakhshan et al. (2010b) reported the results of in-situ out-of-plane tests performed on as-built walls of different URM buildings in New Zealand. The comparison of the aforementioned laboratory and in-situ out-of-plane testing suggested that different wall boundary conditions significantly affected the URM wall behaviour.

The NZSEE (2006) guidelines for seismic evaluation of structures suggest that out-of-plane URM walls can be analysed as either one-way spanning or two-way spanning elements, but only general analysis recommendations are provided for two-way spanning walls. However, the guidelines propose a codified procedure for out-of-plane assessment of one-way URM walls on the basis of the cracked wall “dynamic stability”. Derakhshan et al. (2009b) showed that certain combinations of parameters used in the aforementioned one-way procedure occasionally produces erroneous wall assessment results. As the partition walls in the subject building had strong corner connections that were capable of developing significant two-way actions, the NZSEE one-way spanning wall procedure was not considered as the best option for partition assessment. As no immediate assessment solution for two-way spanning wall evaluation was available, conducting in-situ testing was identified as the best method for the seismic assessment of the partition walls.

An ongoing structural upgrading project of the subject building provided an in-situ testing opportunity, and a series of out-of-plane tests was conducted by a team of student researchers from the University of Auckland. The team mobilized to the building site in two stages, with each stage comprising out-of-plane tests being made on two URM partition walls wtih uniform pressure being applied by airbags. During stage 2 similar testing was conducted on two top-storey URM partitions, and in addition material properties were obtained by extracting masonry prisms from a third URM partition. The results of the in-situ and laboratory testing of the extracted samples from stage 1 and stage 2 are reported, and discussion on wall seismic evaluation is provided.

2.0 BUILDING DESCRIPTION

Weir House consists of three main buildings. The William Weir (WW) wing was built in 1932 as the original structure of the complex (Figure 1), which currently serves as student accommodation for Victoria University of Wellington. This wing is comprised of a main rectangular block (Figure 2) that has footprint dimensions of 56 m by 11 m, and is primarily a 3-storey building with the front section on the eastern side containing a part basement level. A 3-storey western wing structure measuring 21 m by 11 m extends to the west, forming the ‘T’ shaped building plan as seen in Figure 2. The WW wing is registered as a heritage building and as such, any strengthening work had to be sensitive to the existing building fabric and be designed in consultation with Wellington City Council.

The original drawings show the perimeter wall structure to be formed from URM material. However, site investigations revealed that the perimeter walls are constructed from 170 mm thick reinforced concrete with exterior plaster and an interior facing with 90 mm terracotta masonry. Internal partition walls are constructed from terracotta masonry lined with cement plaster. The wall structures are typically punctured with windows and respond with frame action to provide the lateral load resisting system in both the long (N-S) and short (E-W) directions.

3.0 SEISMIC DEMAND

The seismic demand was calculated (Table 1) for the partition walls using the current seismic loading standard, NZS 1170.5:2004. In wall strength demand calculations, a ductility factor of 1.0, an annual probability of exceedance of 1/500, a risk factor of 0.9 (NZS 1170.5:2004 requirements for parts and components, Table 8.1, category P.3) and a soil type of B were assumed. The latter value was based on the available geotechnical records. The masonry density was calculated in a laboratory as 1650 kg/m^3, and this value was used in wall weight calculations. Each partition wall was also evaluated using the NZSEE 2006 recommendations, and %NBS was calculated (Table 1) using the detailed seismic assessment procedure recommended in the guidelines for one-way URM walls.

The NZSEE 2006 assessment results suggested that none of the partition walls in the building satisfied legal New Zealand requirement based on DBH (2004).

Table 1. Wall seismic demand and wall assessment

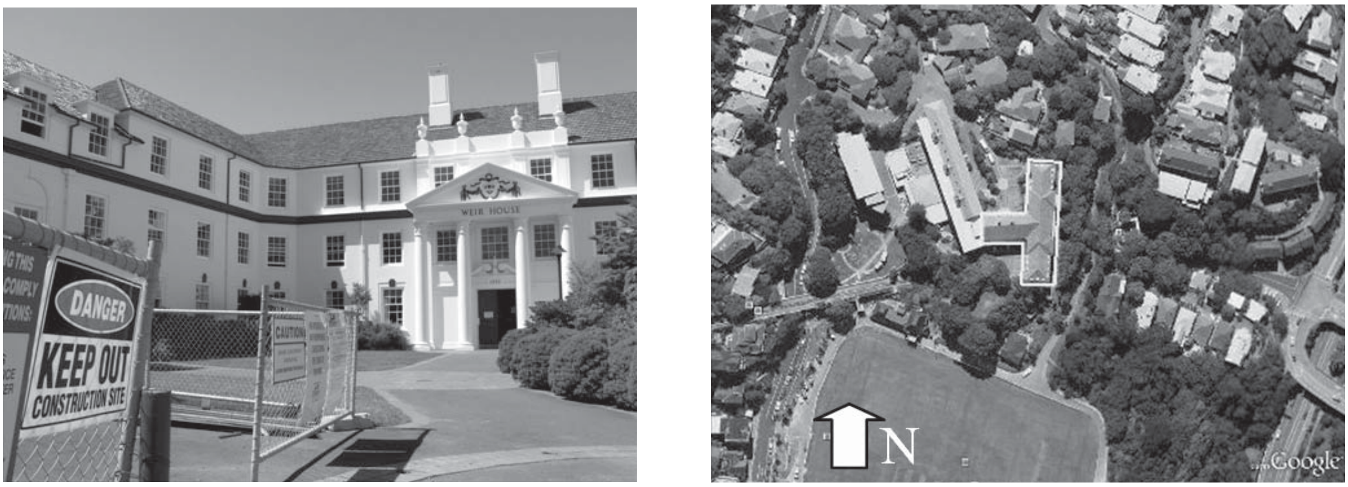
Wall	Strength demand (NZS 1170.5:2004) (kPa)	%NBS (NZSEE 2006)
Building level 2	2.8	23%
Building level 3	3.5	14%

4.0 TESTING PROGRAMME

The field testing programme included 5 static airbag tests conducted on four different partition walls as detailed in Table 2. The tests accounted for as-built wall conditions and wall retrofit using near surface mounted (NSM) fibre-reinforced polymer (FRP) strips, and the airbag tests were conducted on a significantly large portion of the wall surface (70%).

Two partition walls (B03-B04 and B16-B17) were located in second storey, and two partition walls (C8-C9 and C16-C17) were located at the top-storey of the building. The top-storey partition walls had minor cracks extended through the entire wall thickness, with the existing cracks being less than 1 mm in width and being barely visible.

Partition walls B3-B4 and B17-B18 had similar boundary conditions, with both walls connected to reinforced concrete elements at one end and to a masonry return wall at the other end and to concrete floors below and above the wall. Similarly, wall C8-C9 was connected (Figure 3) to a reinforced concrete column at one end, to a masonry return wall (2200 mm long) at the other end, and was constructed on a reinforced concrete floor. Wall C16-C17 had similar boundary conditions to C8-C9, except that the reinforced concrete column was replaced by a reinforced concrete shear wall and a timber wardrobe that contributed to the wall end support as shown in Figure 4.



(a) Street view (facing S-E) **(b) Aerial view (WW wing is outlined)**

Figure 1. William Weir House

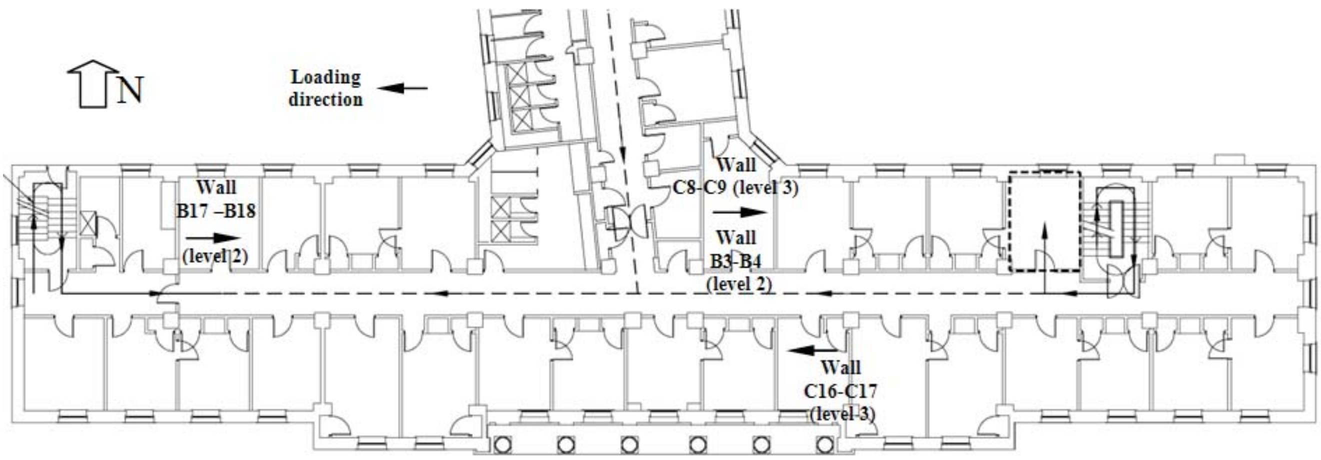


Figure 2. WW wing building plan – level 3 (provided by Dunning Thornton Consultants Ltd)

Table 2. Wall properties and test matrix

Wall	Test code	Partition wall condition	Nominal thickness[a] t_n(mm)	Clear height h (mm)	Length l (mm)	Plaster[b] thickness (mm)
B3-B4	AB1	As-built	125-130	3600	4100	15-20
B17-B18	AB2	Retrofitted	125-130	3600	4100	15-20
C8-C9	AB3	As-built	125-130	2730	3850	15-20
C16-C17	AB4 AB5	As-built Retrofitted	125-130	2730	3480[c]	15-20

[a]Including plaster and rendering [b]Cement-based plaster
[c]Excluding wardrobe length

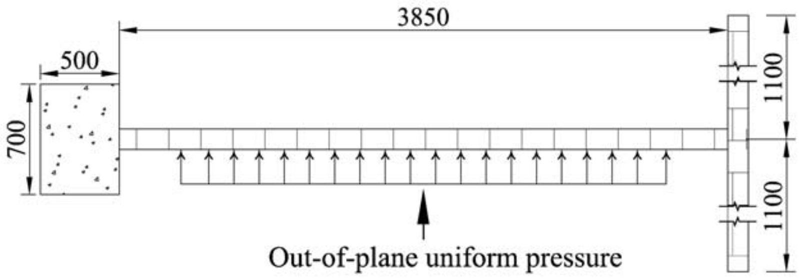


Figure 3. Wall C8-C9 plan

Top-storey walls had a top support condition different from the second level walls. This was investigated by accessing the building roof, and it was found that the wall top was not directly connected to the roof timber structure (Figure 4b). Little or no stiffness was presumed to be provided by the skirting connecting the wall to the ceiling constructed from gypsum plaster boards. During roof inspection on top of wall C8-C9, evidence was found of steel wires (Figure 5a) inserted into the concrete column to secure the masonry wall. The masonry corners lacked full inter-connection in all courses, but a few of the visible top courses (approx. 50%) were interlocked (Figure 5b).

5.0 MATERIAL PROPERTIES

The walls were constructed from dark red hollow clay bricks and cement mortar, with plaster on both surfaces as shown in Figure 6a. The masonry units measured on average 160 mm x 300 mm x 95 mm, and the average mortar joint thickness was 15 mm.

Visual inspections revealed that the mortar had a very high cement content as implied from the mortar being a dark green colour. In addition, the mortar joints maintained a strong bond between the adjacent bricks during masonry prism extraction from a partition, and the brick-mortar bond was judged to be much stronger than had been previously observed by the research team in unreinforced masonry buildings. The plaster had a grey colour and was judged to be less strong than the mortar used between brick courses. In order to perform a subsequent masonry property investigation in the laboratory, 8 masonry prisms were extracted from a partition that was scheduled for demolition. The partition was divided into several segments (Figure 6) each being the size of two masonry units and measuring approximately 330 mm x 300 mm x 130 mm, and a concrete cutting chainsaw was used to cut the samples. The masonry density was calculated from laboratory tests to be 1650 kg/m^3, and the results of the investigation of other material properties, performed in the University of Auckland laboratory facilities, are summarised in Table 3.

The masonry prism compressive strength (f'_m) was determined following ASTM C 1314-03b (ASTM 2003), while the masonry flexural bond strength (f'_{fb}) was derived from four point bending beam tests. The testing processes are shown in Figure 7. The compression test and flexural bond test samples were tested without and with plaster on both sides, respectively. The failure mode

Table 3. Material properties

Property	No. of samples	Average	CoV
f'_m (MPa)	4	13.8	0.50
f'_{fb} (MPa)	3	0.8	0.26
f'_p (MPa)	7	3.4	0.15
EI (kNm^2)	3	2.3	0.24

of the masonry prisms tested in compression was brittle and was mainly initiated by the propagation of vertical cracks through the web-flange joints (see Figure 7a). The four point bending tests were performed with two point loads spaced at 130 mm about the centre of the test specimen, and the roller supports were located at 300 mm apart. The mid span deflections were measured using an LVDT device attached on the test machine. One of the flexural bond test samples failed through the brick web instead of the mortar joint, and yet it was the weakest among the total of three samples tested. The remaining samples failed through the mortar joints and the flexural bond test results confirmed that the brick-mortar bond of this building is much stronger than those of other buildings (Derakhshan et al. 2010b) investigated by the research team. The plaster compressive strength (f'_p) was determined through compression tests of irregular plaster samples (average length, width and height of 26 mm, 21 mm and 26 mm respectively), and the measured mid span deflections were used to estimate the flexural rigidity property of the masonry prisms (EI) using elastic beam bending theories.

6.0 TEST SETUP AND INSTRUMENTATION

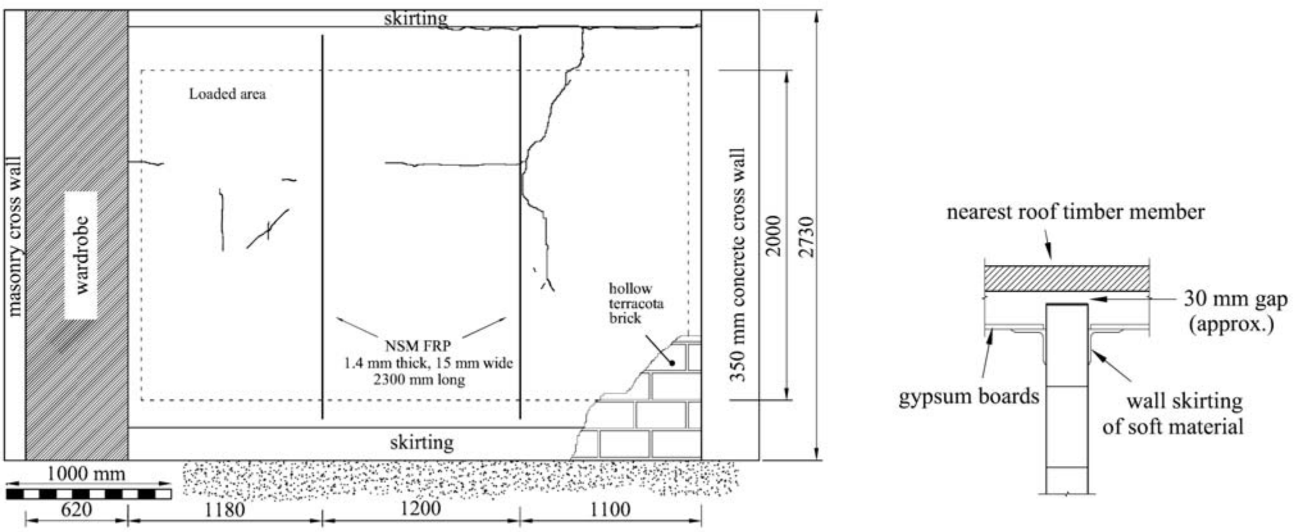
The testing programme included preparation of the airbag testing setup on site and relocation of the setup to two different rooms in the building. The wall surface was first prepared by partial removal of the wall attachments and wiring ducts.

The test setup (Figure 8) consisting of a timber backing frame and timber diagonals, which were assembled and fixed to the concrete floor. Bigfoot™ vinyl airbags with a skin thickness of 0.25 mm were placed in a gap of 100 mm between the wall surface and the backing frame and gradually inflated to generate out-of-plane uniform forces on the wall surface.

(a) Partition wall cross section

(b) Masonry unit interlocking at a wall corner

Figure 5. Photos taken during building roof inspection



(a) Elevation

(b) Top wall edge connection details

Figure 4. Wall C16-C17

The force was transferred through seven 10 kN load cells from the backing frame to timber diagonals before finally being transferred to the floor. In order to ensure that the entire force was transferred through the load cells, and not through the connection between the plywood and the floor, steel plates with a film of grease were used at the backing frame base. Further details of the airbag test setup are provided in Derakhshan et al. (2010a).

By adopting a low inflating rate and selecting the same type and length of hoses and connections for each airbag, it was ensured that multiple airbags were inflated simultaneously to an equal pressure level. A total of four LVDTs and four portal strain gauges were used to measure the relative wall displacement, and the measuring devices were arranged having three vertical axes, with one being along the wall centre (Figure 8b). A National Instruments high-speed data acquisition system with multiple channels (Figure 9) was used to record the test data.

7.0 TESTING

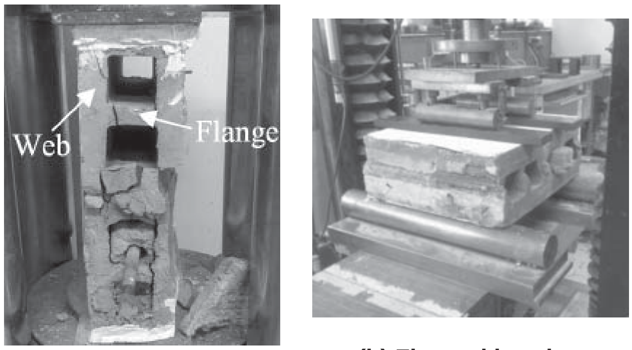
7.1 Stage 1 as-built testing

Due to the limitations of the data acquisition system in stage 1, the force measured by load cells was not recorded above a certain value. Even though it was estimated that the testing continued until 7-8 kPa, a maximum face pressure of approximately 4 kPa was recorded. At the maximum loading pressure no evidence of wall movement at the boundaries and no cracks were visually detected on the wall surface. Due to a potential explosive brittle failure of the wall, attributed to the sudden release of potential energy stored in the airbags, the airbag pressure was not further increased. Consequently, the obtained results in stage 1 were all in the elastic range.

The first test (AB1) was conducted on an as-built partition (Wall B3-B4). The pressure in the airbag was slowly and uniformly increased and the total force exerted by the airbags onto the wall was calculated by summing the readings from the load measurement devices. For comparison purposes the pressure acting on the wall was determined using the total force and the respective loading area.

(a) Partition wall cross section **(b) Masonry prism extraction**

Figure 6. Partition walls



(a) Prism compression test **(b) Flexural bond test**

Figure 7. Material testing

(a) Airbag backing and supporting frame **(b) Wall instrumentation**

Figure 8. Test setup

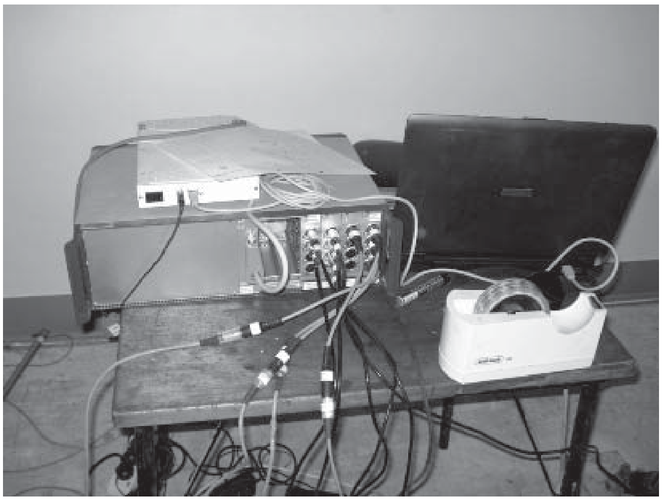


Figure 9. Data acquisition system

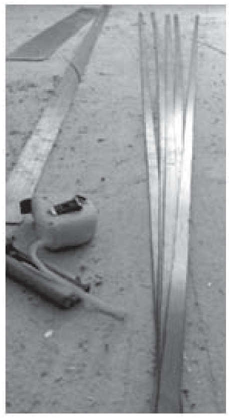
7.2 Stage 1 wall retrofit

Due to time constraints the testing of the as-built partition wall and then the retrofit of the same wall was prevented, and instead a second partition wall (B17-B18) in level 2 was retrofitted and tested (AB2).

Using FRP material to retrofit URM walls is a technique used to strengthen and increase ductility of walls subjected to in-plane and out-of-plane earthquake loading. Externally bonded (EB) FRP sheets or plates and NSM of FRP bars or strips are the two application techniques that are commonly used (Mosallam 2007, Yasser et al. 2006). Using the NSM technique provides some protection from fire and the environment and if detailed correctly, does not adversely affect the aesthetics of the structure (Petersen and Masia 2008). In this field experiment NSM FRP strips as a seismic retrofit intervention of URM walls was investigated.

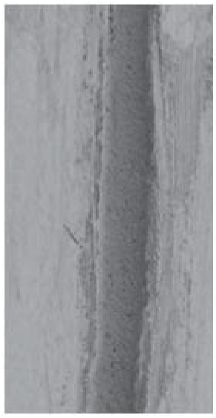
Wall B17-B18 was reinforced using CFRP strips (Young Modulus equal to 165 GPa) 15 mm wide and 1.4 mm thick, as shown in Figure 4. Grooves were cut into the masonry with a circular saw (5 mm thick blade) to a depth of approximately 30 mm as shown in Figure 10(a), with the depth selected to ensure direct bonding of CFRP strips to the brick surface and not the plaster layer. Two part epoxy adhesive was used to bond the CFRP strips into the grooves in the brick. The grooves were entirely filled with epoxy prior to CFRP strip insertion to ensure maximum bond area. Figures 10(c) and (d) show the installation procedure.

(a) Groove cutting



(b) 15 mm CFRP strips

(c) Epoxy application



(d) Groove pointing

Figure 10. NSM CFRP installation

Figure 11 shows a comparison between the force-displacement plots of wall B3-B4 (as-built, AB1) and wall B17-B18 (retrofitted, AB2). Surprisingly, the flexural stiffness of the retrofitted wall was less than that of the as-built wall. Because of identical dimensions for wall B3-B4 and B17-B18, the difference in flexural stiffness of the walls was attributed to the possible variation in material properties and difference in quality of construction between the walls. Consequently, the flexural stiffness varies from wall to wall within the building.

Figure 11 shows that both walls satisfied the current seismic demand (NZS 1170.5:2004), estimated to be approximately 2.8 kPa.

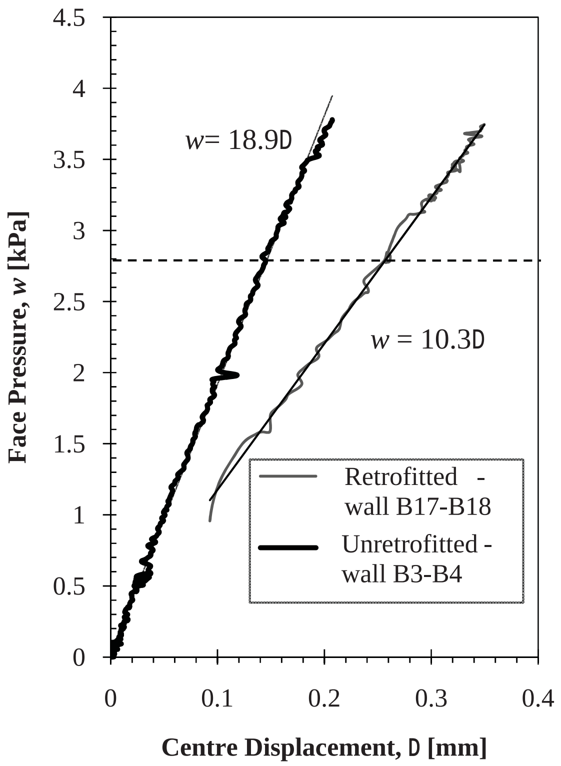


Figure 11. Response of as-built and retrofitted walls in stage 1

The lateral out-of-plane response of wall C16-C17 is shown in Figure 14 for the performed as-built and retrofitted tests. Analysis of the response envelope in Figure 14 showed that at a face pressure of approximately 3.2 kPa the wall stiffness reduced by approximately 65% from 3.6 kPa/mm to 1.2 kPa/mm.

Both walls underwent small amounts of plastic deformation (approx. 0.5 mm), and both walls satisfied the current seismic demand (NZS 1170.5:2004) as depicted in Figures 13 and 14.

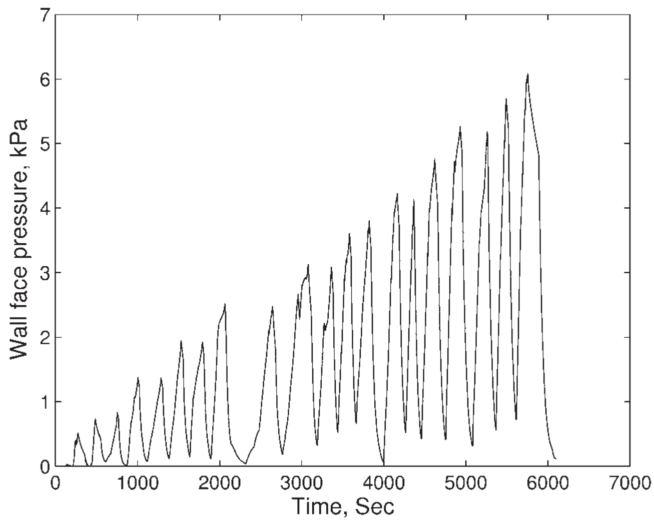


Figure 12. Typical half-cycle loading history

7.3 Stage 2 – as-built testing

In stage 2 testing, test AB3 was conducted by applying load in 2 half-cycles, but a typical loading history as represented by Figure 12 was applied in test AB4 and AB5. The first test, AB3, was conducted on wall C8-C9 in the as-built condition. Figure 13 shows the equivalent wall face pressure versus the wall mid-height displacement. The wall face pressure was calculated as the total measured lateral force sensed by the load cells divided by the wall total area. As mentioned previously, a crack pattern appeared to have existed on the top-storey partition walls before commencement of the tests. Although monitoring the crack propagation was difficult due to a layer of paint on the wall, subsequent crack pattern observations confirmed that the wall damage remained limited to the existing cracks being opened. At an equivalent wall face pressure of approximately 1.4 kPa, a few vertical cracks were visible closer to the top (unrestrained) edge of the wall, and thereafter the crack pattern and extent of crack widths remained constant. The wall face pressure exceeded the current seismic load demand as shown in Figure 13, and the wall capacity up to 149% of the calculated demand (5.2 kPa vs. 3.5 kPa) was confirmed.

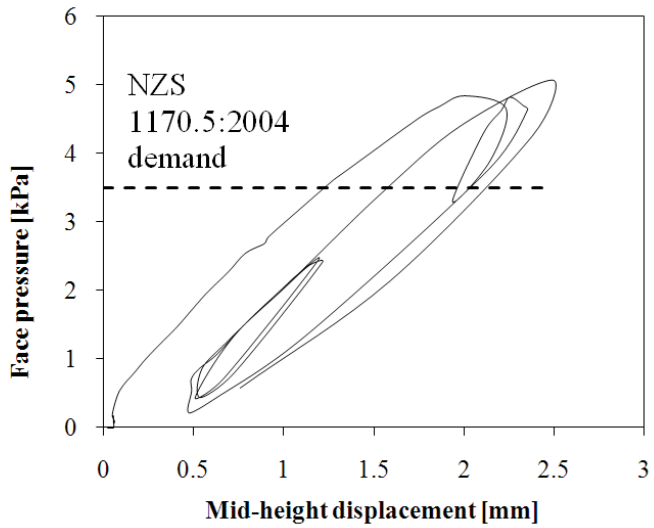


Figure 13. Test AB3, wall C8-C9

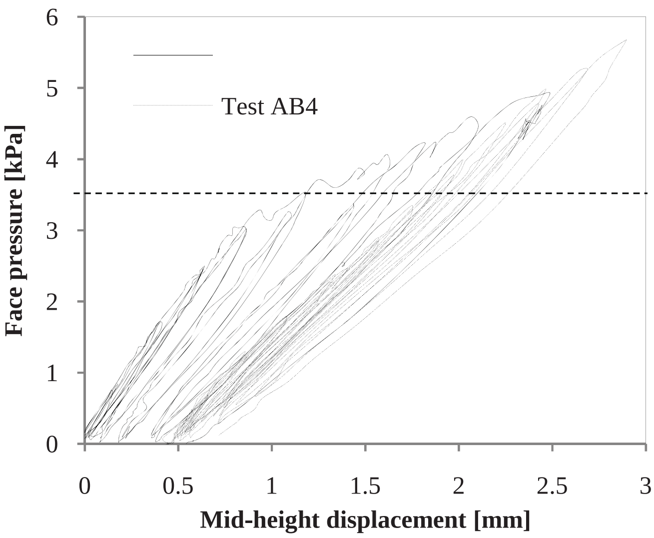


Figure 14. Test AB4 and AB5, wall C16-C17

7.4 Stage 2 – wall retrofit

Wall C16-C17 was retrofitted using a similar techniques to that described in previous section and it was re-tested. The results of the retrofitted wall test are shown in Figure 14. It can be seen that the flexural stiffness of the partition wall increased following the retrofit from 1.2 kPa/mm to approximately 1.8 kPa/mm.

8.0 DISCUSSION OF THE TEST RESULTS

Table 4 compares the URM partition wall behaviour with the strength demand calculated using the NZS 1170.5:2004 (NZS, 2004) standard and also presents the results of the wall evaluation independently made using the NZSEE (2006) document for seismic assessment of buildings.

The tested partitions of WW building demonstrated significant resistance, and the maximum applied face pressure was approximately 30 times higher than that expected from a one-way wall with the same thickness (calculated as 0.2 kPa using an empirical relationship from Derakhshan et al. 2010a).

Table 4 suggests that all walls satisfied the strength demand requirements based on the current seismic loading standard. The observed minor cracking in top-storey walls was considered acceptable, as the walls remained stable after testing, with no loss of flexural strength being observed.

The wall assessment assuming one-way behaviour and based on the NZSEE 2006 document suggests that none of the walls meet the minimum legal requirements with respect to earthquake prone building policy in New Zealand (%NBS<33). Based on the experimental observations summarized in Table 4, which accounts for wall capacity of at least 149% and 166% of the current seismic demand for different walls, the one-way evaluation procedure is considered excessively conservative.

Retrofit test in stage 1 was conducted on a partition wall, which was not previously tested in as-built condition; therefore effectiveness of retrofit was not evidenced by direct comparison of the test results. In stage 2, the effectiveness of a retrofit scheme was shown by comparing the test results. Although the change in wall behaviour was not significantly high, other in-situ testing of this retrofit technique has shown significant increase in flexural strength of out-of-plane loaded URM walls (Dizhur et al. 2010a, Dizhur et al. 2010b, Dizhur et al. 2010c). Even though further research is required to accurately establish the design guidelines for this type of retrofit, this and previous testing has illustrated that the NSM retrofit technique provides a simple and cost effective alternative to seismically strengthen URM buildings and their components.

The testing programme in both stage 1 and stage 2 showed that the strength of all URM partitions exceeded current NZ seismic demand calculated using NZS 1170.5:2004. This indicated that no seismic strengthening was required for partition walls, and resulted in significant financial benefits for the building owner.

Table 4. Summary of the test results

Wall	Maximum wall face pressure (kPa)	Strength demand (NZS 1170.5:2004) (kPa)	%NBS (NZSEE 2006)
B3-B4	3.8	2.8	23%
B17-B18	3.8	2.8	23%
C8-C9	5.2	3.5	14%
C16-C17	5.8	3.5	14%

9.0 CONCLUSIONS

In-situ tests were conducted on three as-built URM partition walls of Weir House in Wellington, and two tests were performed on retrofitted walls using NSM FRP technique. The walls developed two-way bending actions and exhibited significant resistance to the applied face pressures, although some of the partitions had undergone previous minor cracking. The maximum measured wall strength was calculated to be approximately 30 times that predicted for a one-way wall, suggesting that the one-way assumption is very the conservative for the tested partition wall.

Comparison of the test results with seismic demand calculations showed that both as-built walls satisfied the current New Zealand seismic loading standard, indicating that no seismic strengthening was required. Independent calculations showed that the procedure recommended in the NZSEE 2006 guidelines for assessment of one-way walls is excessively conservative, if used for the tested two-way walls.

As two separate specimens were tested in stage 1, it could not be concluded if the flexural stiffness of the wall was affected by the NSM CFRP retrofit. It has been shown by testing in stage 2 that CFRP strips embedded into the wall provided an increase in flexural stiffness. Even though further research is required to accurately establish design guidelines for this type of retrofit, this and previous testing has illustrated that this retrofit technique provides a simple and cost effective alternative to seismically strengthen URM buildings and their components.

The testing of walls that were laterally supported along three or four edges highlighted the need to develop assessment guidelines for walls that are capable of developing two-way actions.

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